Best Practices in Dam and Levee Safety Risk analysis

20. Concrete Gravity Structures

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Concrete Gravity Structures

- Background Information
  - This section is intended to provide an overview of information related to risk assessment considerations for mass concrete gravity structures
  - The focus will be on traditional stability analyses, but with key points related to appropriate modifications for risk analyses
  - Mass concrete gravity structures are generally very reliable, particularly if designed with proper assumptions/safety factors and good construction practices
  - Most known failures related to mass concrete gravity structures are attributed to foundation issues
    - Sliding along a weak plane or fault (founded on rock)
    - Piping of alluvial material within the foundation (soil founded)

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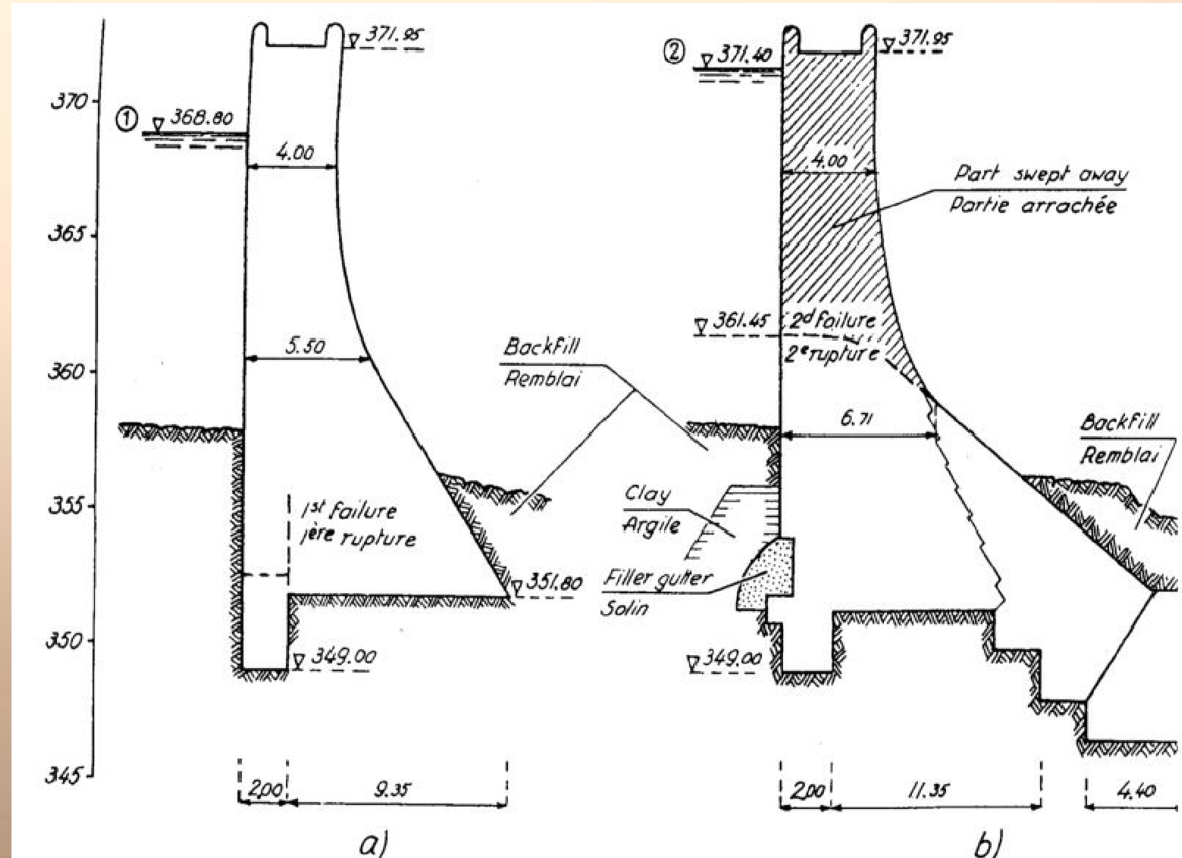
Concrete Gravity Structures

- Case Histories of Failures/Significant Damage
  - Bouzey Dam, France – 1884
    - Structural failure through the upper section of the dam
  - Elwah Dam, Washington – 1912
    - Piping of alluvial foundation
  - Austin (Bayless) Dam, Pennsylvania – 1911
    - Sliding along weak foundation plane
  - Koyna Dam, India – 1976
    - Significant structural damage, but no breach (earthquake damage)

Bouzey Dam, France (Structural)

- 72’ high masonry gravity dam built in 1884
- Structural damage on initial filling included shearing of key, but no vertical significant vertical displacement when water reached 10-ft from crest
- Other damage during initial filling included cracking along upstream heel of the dam
- D/S lower third of the dam was strengthened by providing a buttress and keying it deeper into the foundation (horizontally-bedded sandstone)
- Subsequent filling in 1895 up to 2-ft from the crest resulted in the upper narrow section of the dam suddenly failing on April 27, 1895
- The failure released a torrent of water on the village of Bouzey causing more than 100 deaths

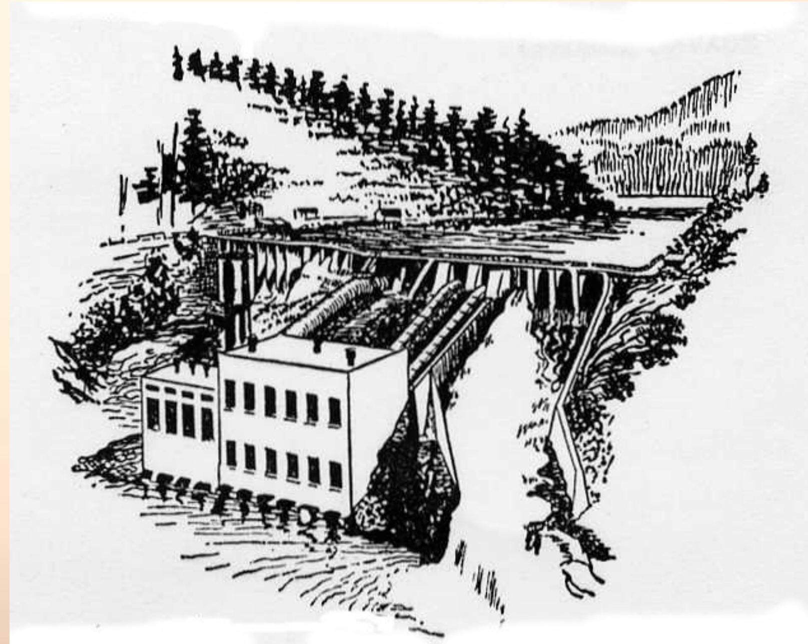


Bouzey Dam

- Tensile crack likely originated at the upstream face during the first filling due to excessive moment at the base of the narrow upper section
- No structural modifications made to upper section of dam, only buttress section added to base of dam
- Crack propagated through the structure and did not have enough shear strength (friction) to resist the driving forces brought on by the 2nd filling (first time to within 2-ft of crest)
- Masonry mortar used dirty sand of poor quality
- Uplift recognized as a contributor for first time
- Horizontal joint opening and subsequent uplift resulted in failure

Elwha Dam, WA

- Constructed in early 20th century
- Originally tried to place upstream cutoff, but had trouble during construction
- No cutoff to rock
- Large seepage flows developed D/S during filling
- Attempted to improve seepage by placing single row of sheet piling 30-ft deep about 8-ft d/s of toe of dam
- Unsure why they chose this location
- Very high exit gradients between toe of dam and sheet piles (base of dam served as 'roof' of the piping failure)



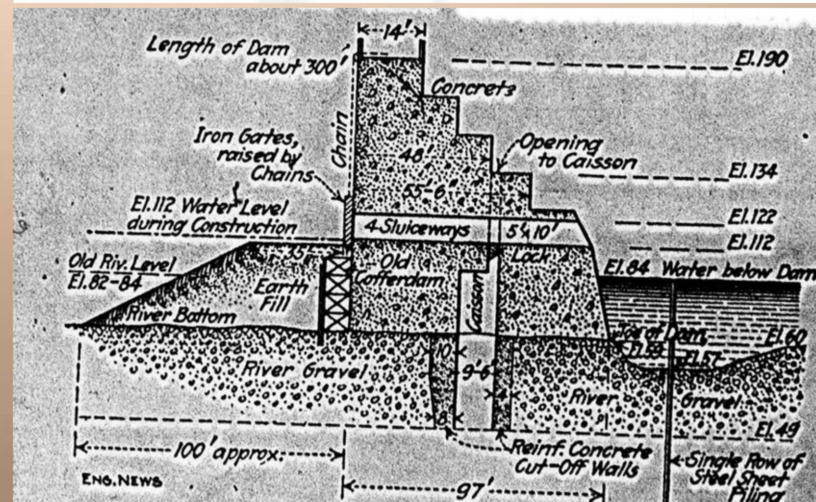


FIG. 1. CROSS-SECTION OF THE PORT ANGELES DAM, SHOWING THE CUT-OFF WALLS WHICH WERE BEING SUNK BELOW THE MAIN DAM WHEN THE LOOSE RIVER GRAVEL BLEW OUT (OCT. 30, 1912)

Elwha (WA) and Hauser (MT)

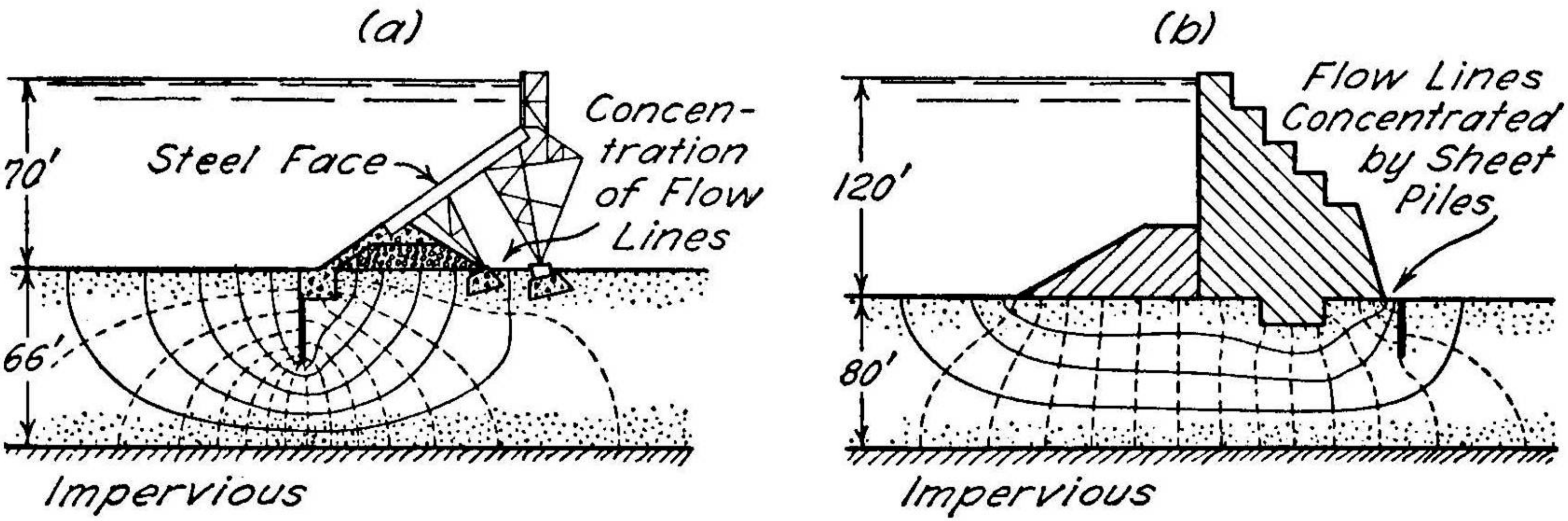
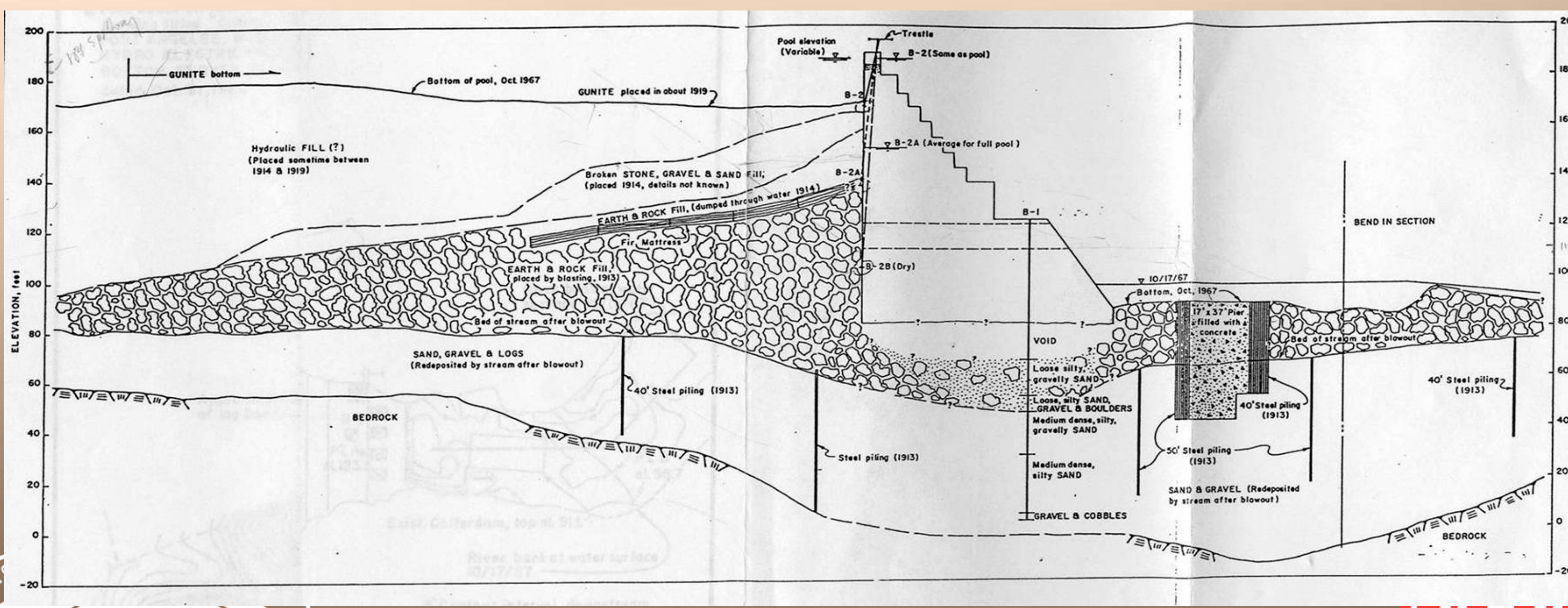


FIG. 208. Flow nets showing concentration of flow lines responsible for failure by piping of two dams; (*a*) Hauser Lake Dam, Mont. (*b*) Elwha River Dam, Wash.

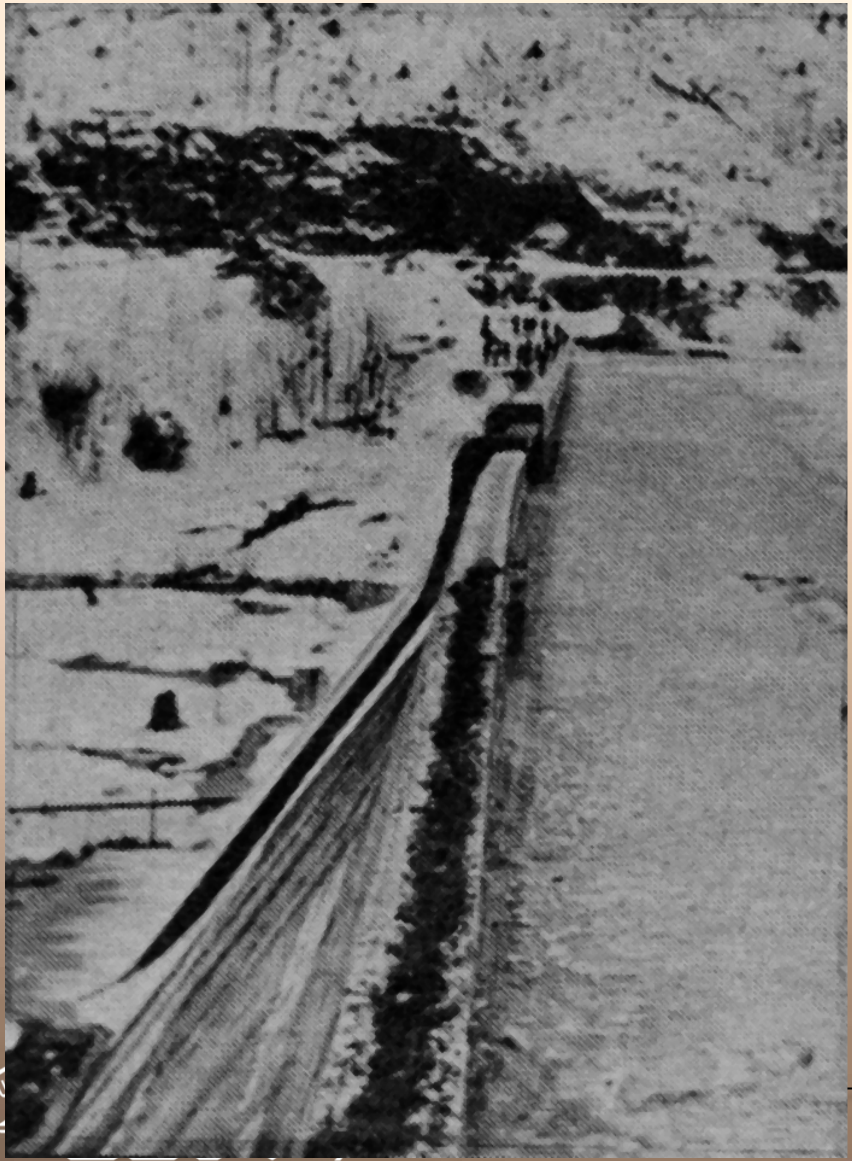
- Terzaghi and Peck noted this failure, and attributed it to concentration of flow lines

Elwha Dam

- October 30, 1912 piping failure of alluvium under dam
- 8,000-12,000 acre-ft passed under dam in 1½-3 hrs
- Main portion of dam on rock abutments spanned hole
- Hole was 75’ deep at U/S face, 90’ deep at D/S face
- Extensive property damage but no life loss
- Extraordinary measures to store reservoir



Austin (Bayless) Dam, PA



- Dam built by Bayless Pulp & Paper Company
- Capacity between 550-850 acre-ft
- 52' high gravity dam constructed approximately 1-1/2 miles above the town of Austin, PA
- Dam constructed on horizontally bedded sandstone with interbedded layers of shale
- January 17, 1910 - center base slid 18" with crest deflection of 31"
- Reservoir lowered after movement
- Repairs recommended after an engineering assessment was completed

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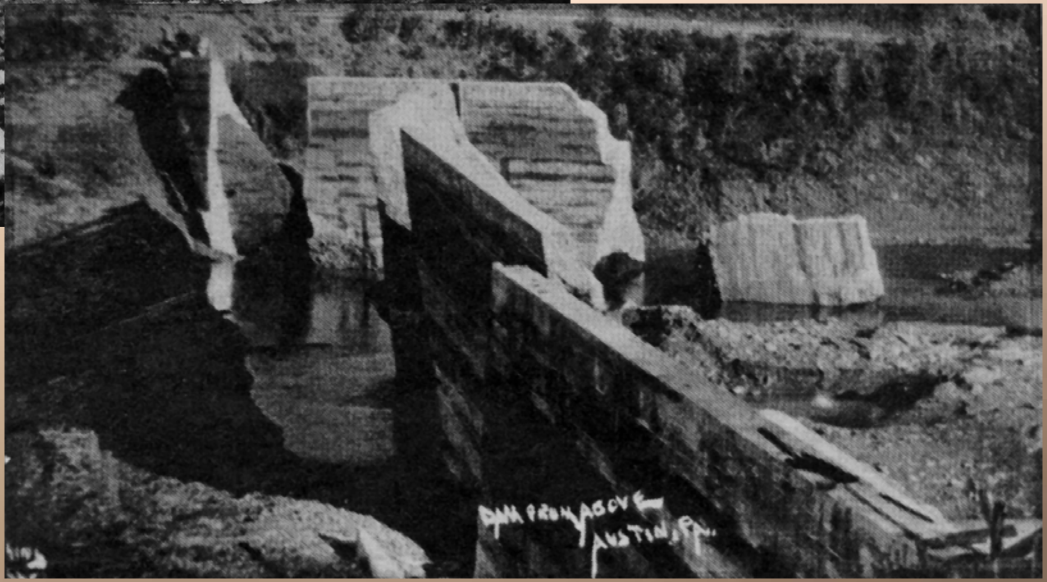
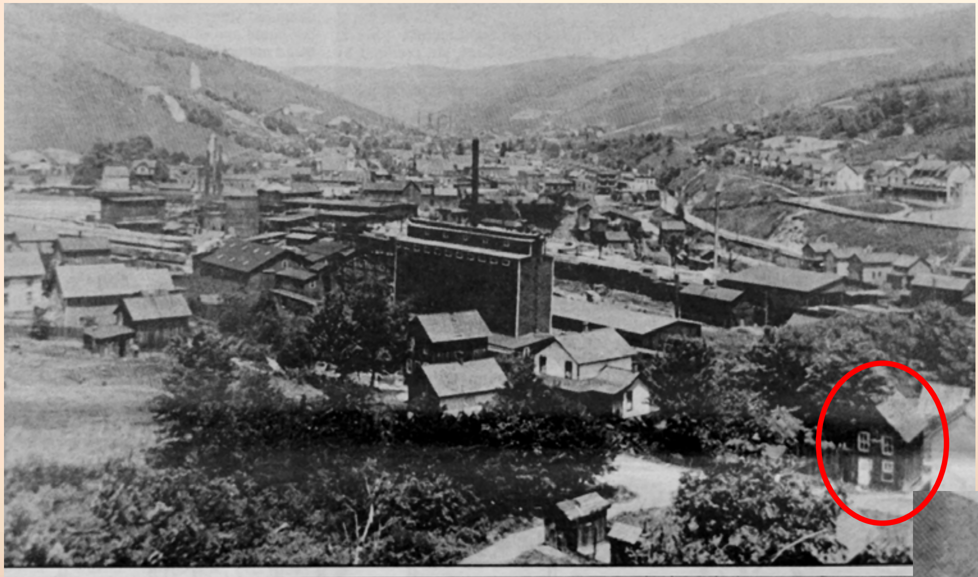
Austin (Bayless) Dam



- Repairs were not made
- Dam failed suddenly Sep 30, 1911 after heavy rains with water 7” deep over spillway
- Eyewitness accounts indicate a plug shaped section near the base of the dam ‘blew out’ and water surged through the opening
- Other sections of the dam opened like a swing gate
- Foundations of some of the failed sections still were attached to the base of the dam indicating sliding along a weak plane deeper in the foundation

Austin (Bayless) Dam



- Paper mill whistle blew but warning went unheeded due to previous false alarms
- Flood destroyed the town of Austin leaving only a few brick buildings and houses located above the flood wave

- Flood wave dissipated by time it reached town f Costello located 3 miles below Austin
- Total of 78 fatalities, all in the town of Austin

Austin (Bayless) Dam

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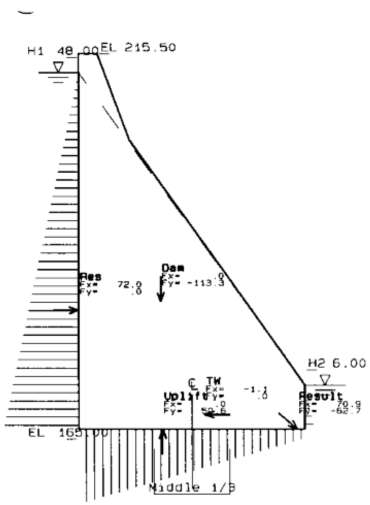
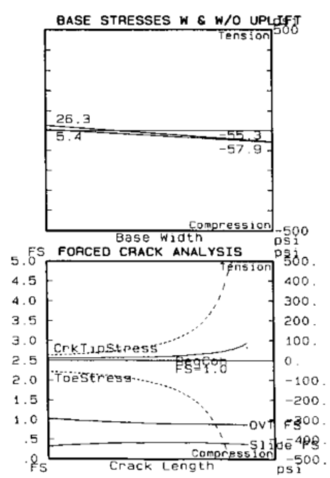
AUSTIN DAM





DAM DIMENSIONS

Analysis type	Standard
Criteria at drains	Corps
Top of dam elevation	215.500
Base of dam elevation	165.000
Thickness of crest	2.500
Base width	30.000
Reservoir elevation	213.000
Tailwater elevation	171.000
Silt elevation	.000
Initial crack length	.000
Drain distance from axis	.000
Drain distance from heel	.000
Drain efficiency	1.000
Drainage gallery elevation	.000
Head at H1, Heel	48.0000
Head at H2, Toe	6.0000
Head at H3, Drain	48.0000
CORPS H3=H1	

MATERIAL PROPERTIES:

Density of concrete		.13500 K/ft3	
	water	.06250 K/ft3	
	horiz sat silt	.085 K/ft3	
	vert sat silt	.120 K/ft3	
Cohesion:	Break bond	.000 lb/in2	
	Apparent	.000 lb/in2	
Friction:	Bonded	.364	20.0 deg
(tangent&angle)	Unbonded	1.000	45.0 deg
Fraction of area bonded		1.000	

CG ABOUT 0.0, FORCE & MOMENT ABOUT CL UNCRACKED BASE

Desc	ForceX	ForceY	CGX	CGY	MomentX	MomentY
Dam	.0	-113.3	4.1	17.2	.0	-465.4
Res	72.0	.0	15.0	16.0	1152.0	.0
Tail	-1.1	.0	-1.7	2.0	-2.3	.0
Uplt	.0	50.6	3.9	.0	.0	196.9

FORCES	w/uplift	w/o uplift
Horizontal (+=d/s)	70.88	70.88 K
Vertical (-=down)	-62.72	-113.34 K
Resultant	94.64	133.68 K
Location from CL	14.05	6.04
Distance 1/3 base	5.00	
Distance 1/2 base	7.50	
MOMENTS		
15.00 from heel	881.22	684.35 k-ft
OVERTURNING SF =	1.03	

VERTICAL STRESSES AT HEEL AND TOE (Tension is positive)

	w/uplift	w/o uplift
Area of base, A		30.00 ft2
Moment of inertia, I		2250.00 ft4
Moment arm, c		15.00 ft
Axial stress, P/A	-14.52	-25.24 lb/in2
Moment stress, Mc/I	40.80	31.68 lb/in2
Vertical stress at heel	26.28	5.45 lb/in2
Vertical stress at toe	-55.31	-57.92 lb/in2

MINIMUM COMPRESSIVE STRESS AT HEEL FOR NO CRACKING
szu = pwh - ft/s
Drain factor, p= 1.000
Safety factor, s= 1.00

Tensile Strength lb/in2	szu Allowable lb/in2	Heel Condition
.00	20.83	Cracked
25.00	.00	Cracked
75.00	.00	Cracked
125.00	.00	Cracked
175.00	.00	Cracked
225.00	.00	Cracked

SLIDING FACTORS OF SAFETY
Driving force 70.88 kips
Total vertical forces -62.72 kips
Safety Factor = .32 for above strengths
Cohesion (Bonded) = 11.12 psi req for FS= 1.0
Sliding safety factors for various cohesions:

BreakBond-psi	.0	50.0	100.0	150.0	200.0	250.0
Resisting-K	22.8	238.8	454.8	670.8	886.8	1102.8
Safety factor	.32	3.37	6.42	9.46	12.51	15.56

Required strengths to get these safety factors while keeping other given values constant

		1.0	2.0	3.0
Bonded area:	cohesion (psi)	11.1	27.5	43.9
	friction angle (deg)	48.5	66.1	73.6

FORCED CRACK RESULTS
Heel initially in tension
Crack extends entire width of base

- Primary cause sliding on weak shale layers within foundation
- Post failure calculations indicated a friction angle < 41 degrees would result in F.S. < 1

Austin (Bayless) Dam, PA

Results courtesy of Brain Greene, USACE Pittsburgh Dist.,
and Daniel Martt & Abdul Shakoor, Kent State University

Material	Cohes-ion (lb/ft^2)	Friction Angle (°)
Concrete over Sandstone	15000	25
Sandstone over Sandstone	5984	31
Sandstone over Shale	0	25
Shale over Shale	3371	25

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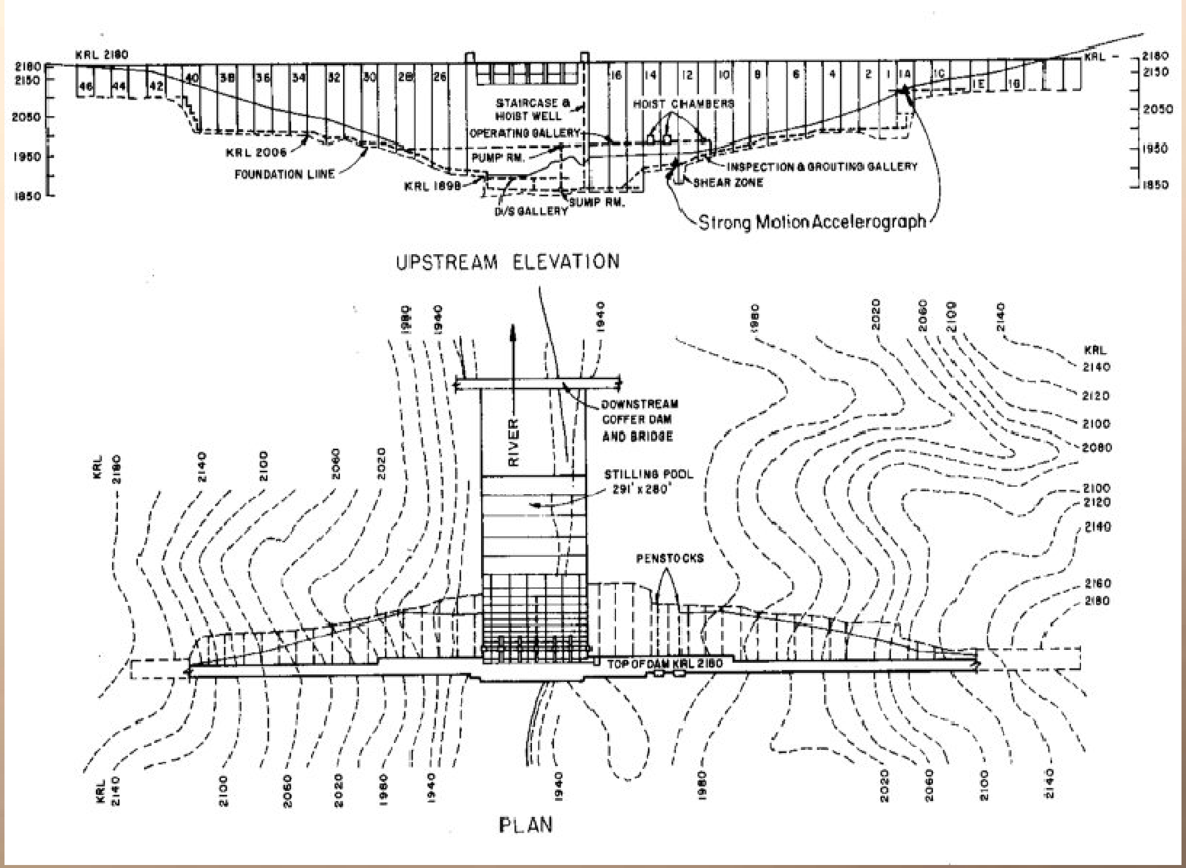
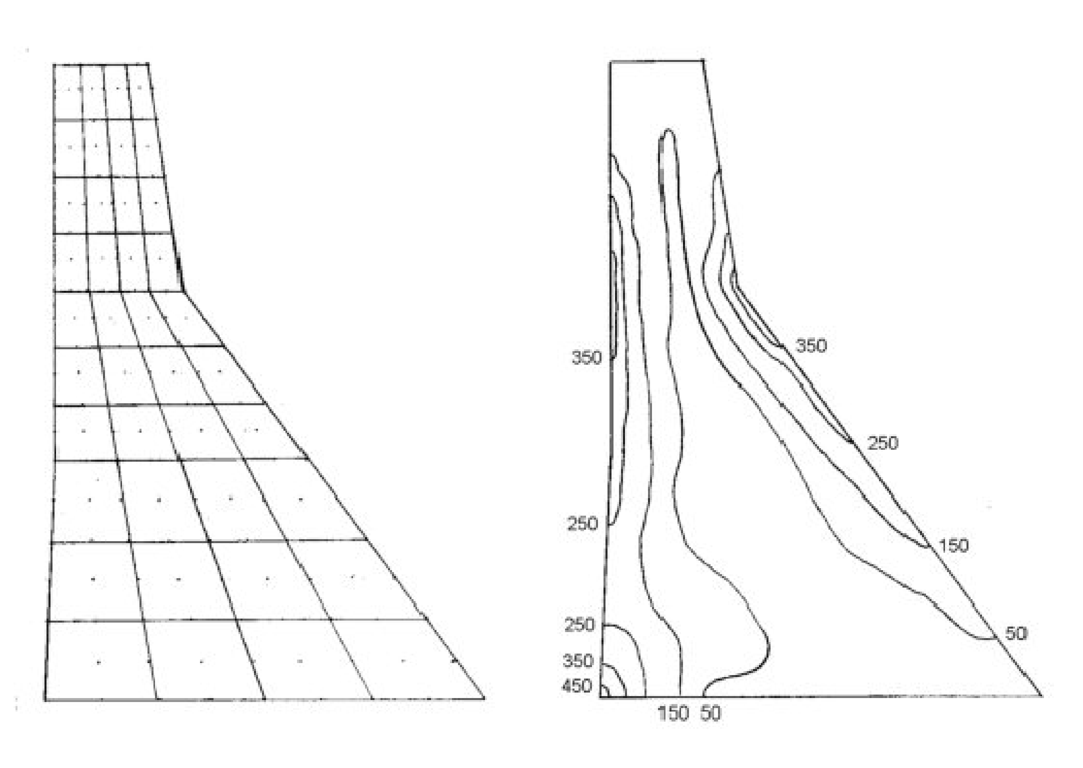
Koyna Dam, India (earthquake)

- Straight axis gravity dam located in SW India
- 340-ft high, 2800-ft long
- 50-ft wide monoliths
- Joints not keyed, but contained copper water seals
- Modifications during construction caused a change in geometry of non-overflow monolith cross section
- Steeper d/s slope near top of monolith



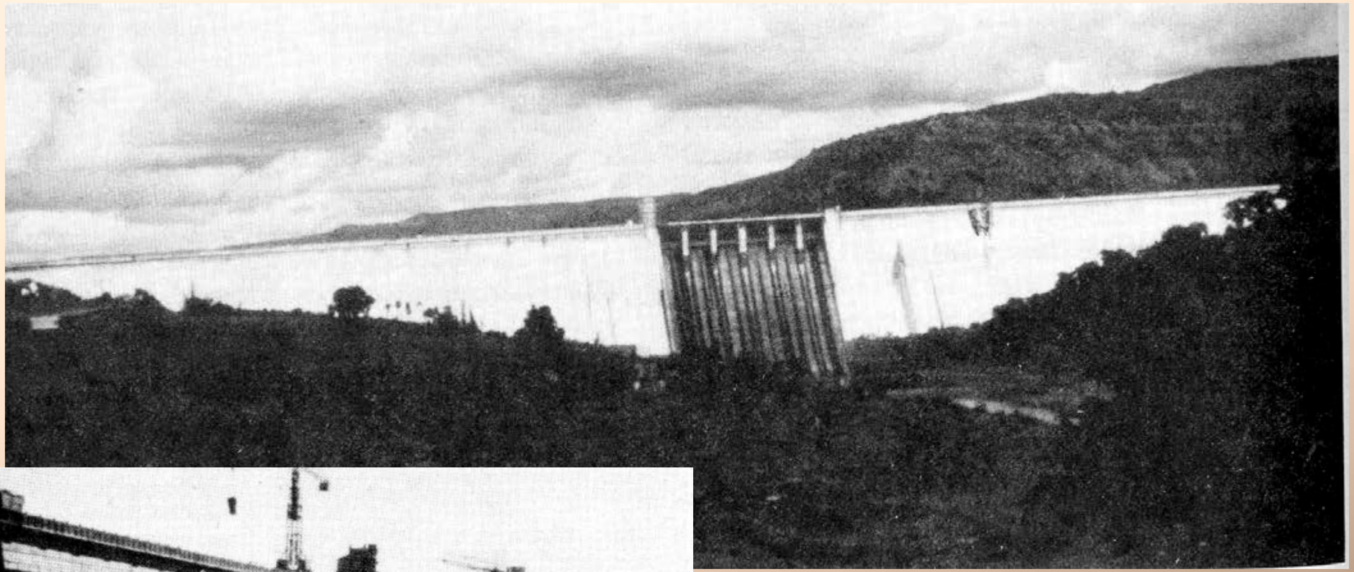
Koyna Dam

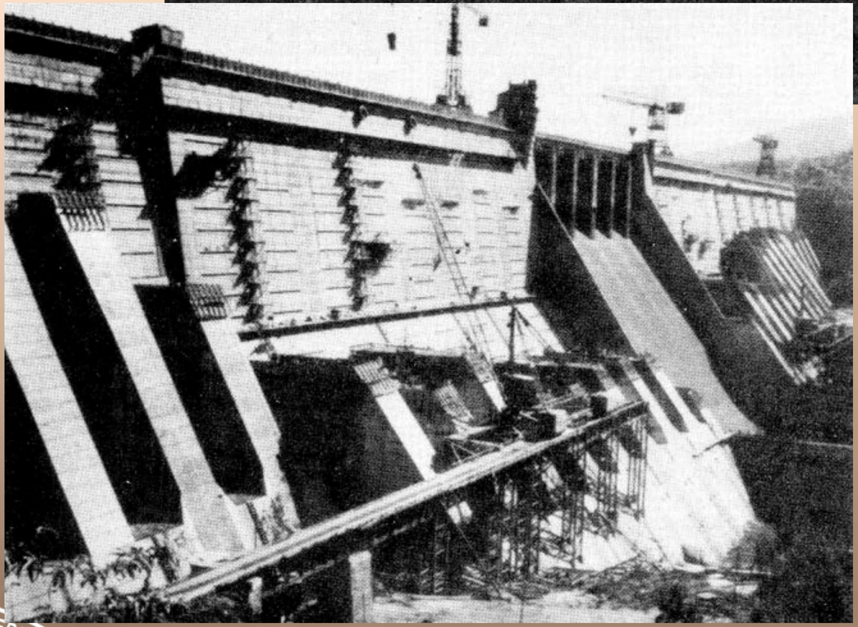
- M6.5 EQ on 12/11/76 with epicenter only 13 km from dam
- Reservoir within 40-ft of crest at time of EQ
- Deep horizontal cracks u/s and d/s faces occurred causing significant leakage in most non-overflow monoliths near change in slope
- Modern linear elastic analysis showed tensile stresses > tensile strength



Results of EAGD analysis

Koyna Dam





Dam strengthened after the EQ

Design versus Risk Analysis

DESIGN CONSIDERATIONS

- Analysis is generally deterministic
- Incorporation of FS for stability analysis
- Lower FS for non-routine loads
- Do not account for side friction
- Typically assume ineffective drains at least as one load case
- Generally assume lower bound values for resisting forces (friction, etc)
- Generally don't consider interlock resistance for monoliths with keys
- FS < 1 is 'ultimate' limit state for design
- Past performance is generally not considered
- May consider 3-D effects and risk-based loading

RISK ANALYSIS CONSIDERATIONS

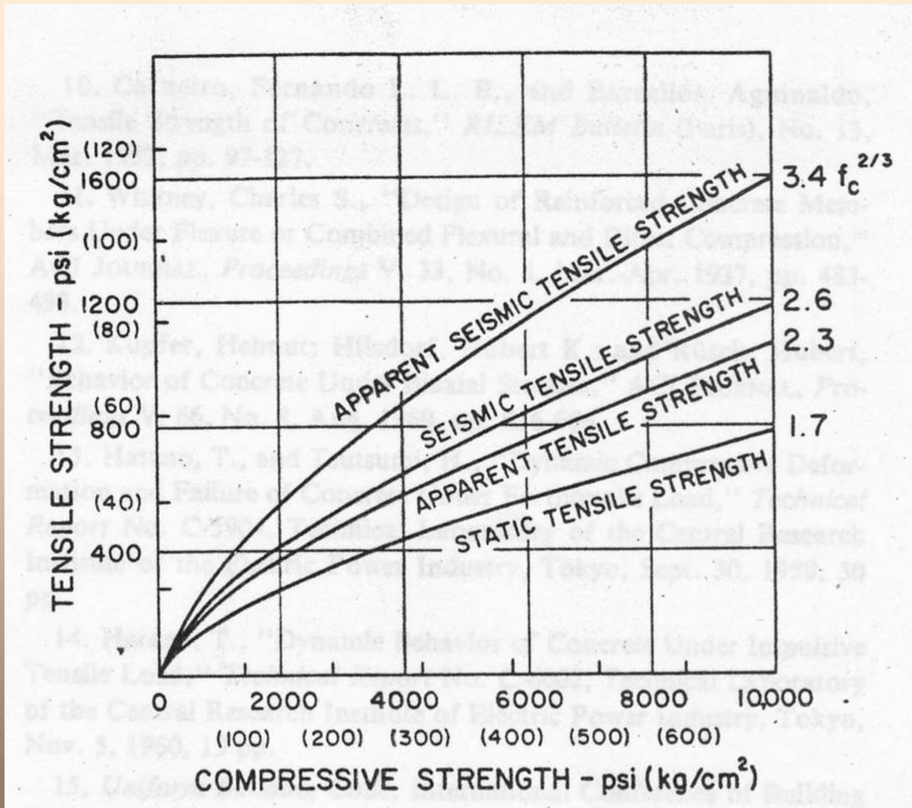
- Analysis is probabilistic
- No safety factors considered
- Account for frequency of loading
- Should try to account for side friction when it is likely to provide additional resistance
- Account for actual drain efficiency with data
- If no data available, use information regarding environment, maintenance, etc. to determine a best estimate
- Full range of values for analysis parameters with best estimates, bounds, and distributions
- FS < 1 associated with a traditional stability analysis is not likely the limit state for RA
- Past performance can be a significant contributor to estimating risks

Key Concepts

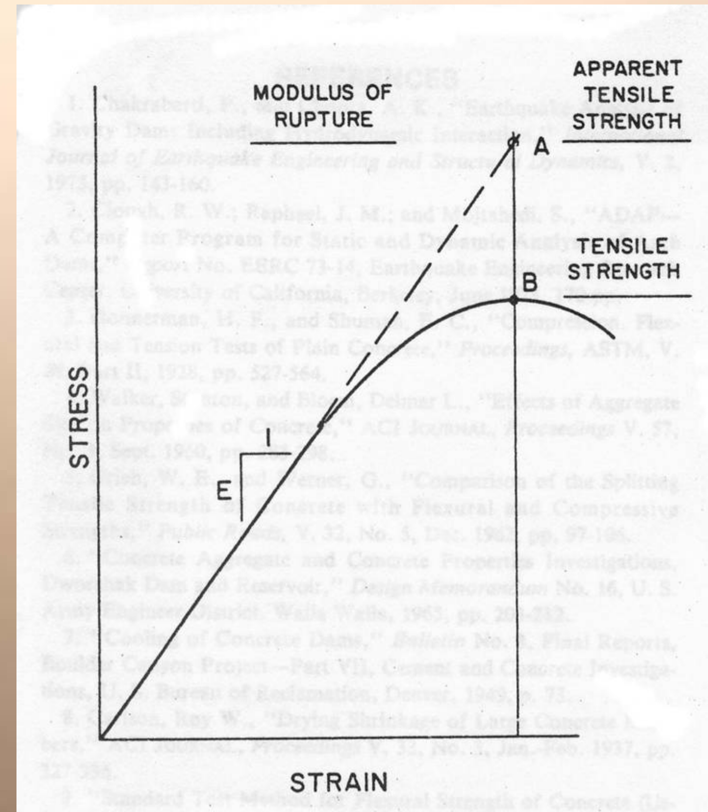
- Gravity dams founded on alluvial foundations – see internal erosion section
- Sliding on weak lift joints or foundation discontinuity key for gravity dams founded on rock
- Foundation interface typically rough due to blasting
- Lift joint clean up and placement practices key to strength of joints
- Line of functioning drains adds to stability
- Shear keys in joints enable load transfer between monoliths

Concrete Tensile Strength

As per Raphael (1984), it is best to use splitting tension tests for best estimate of tensile strength of concrete



Raphael (1984) suggests a 50% increase in tensile strength for dynamic loading



May use apparent strength if a single spike is considered to cause cracking

Concrete Tensile Strength

- In depth evaluation by Bob Cannon (1995) confirmed that splitting tensile strength are a good starting point
- Adjustments for large size aggregate (10% reduction)
- Adjustments for direct tension and anisotropy (20% reduction)
- Confirmed a 50% increase for dynamic tensile strength
- Recommendations for RCC
- See Corps of Engineers EP 1110-2-12, 30 Sep 95, Appendix E
- Information can be used to estimate likelihood of cracking from calculated stress results
- Not valid for ASR-affected concrete

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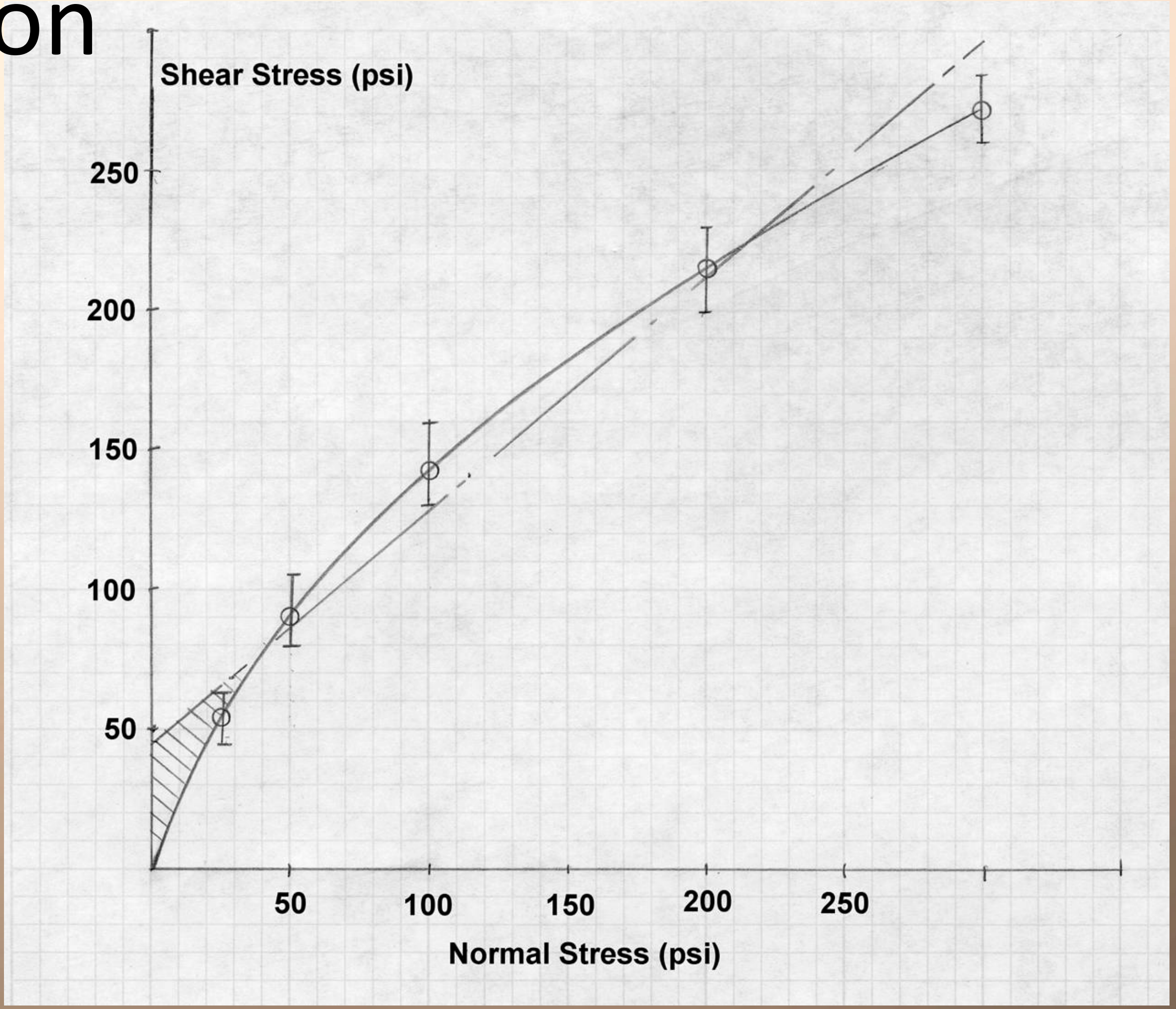
Cracked Base Analysis

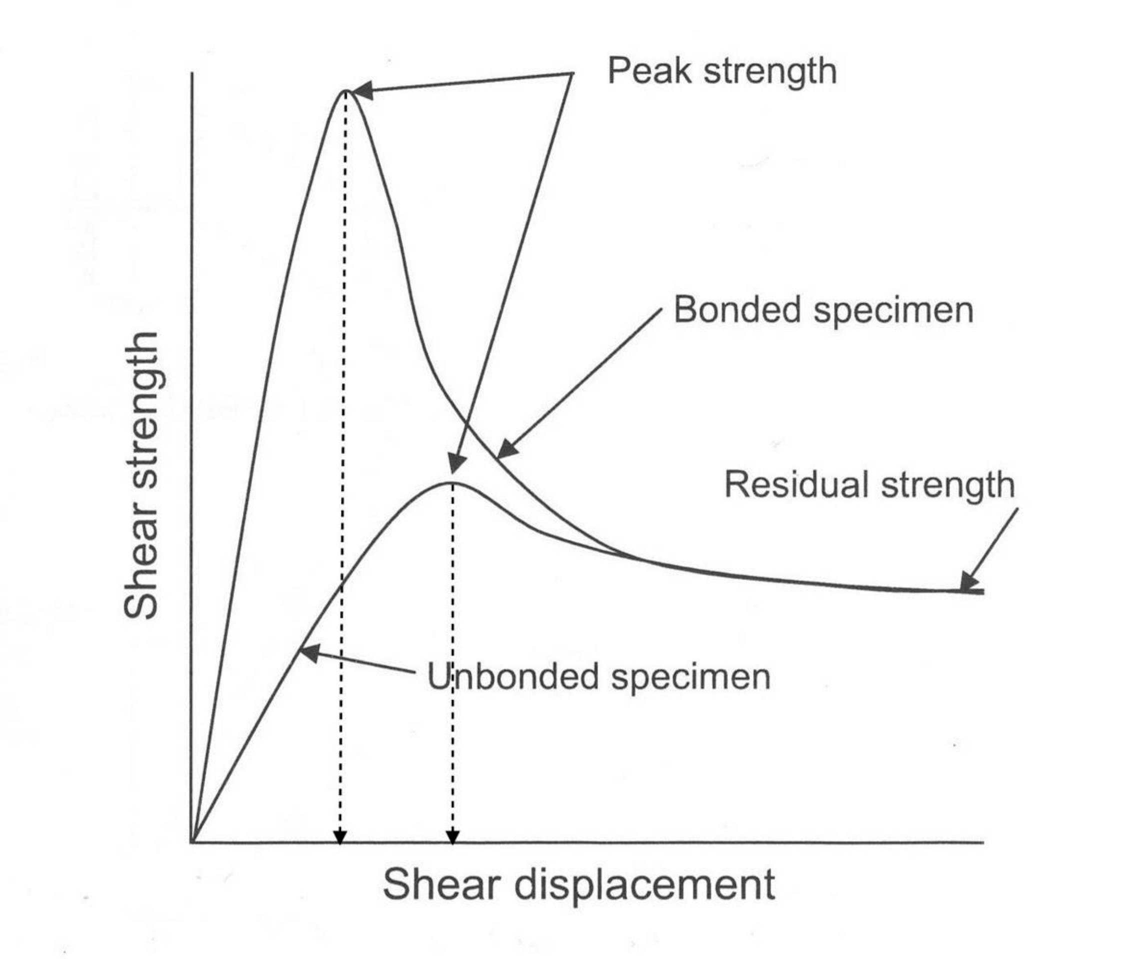
- Most published methodology and criteria are geared more towards design and are generally too conservative for risk analysis purposes
  - Full uplift at crack tip for most concrete dams is not reasonable due to the fact that the foundation permeability > permeability of the crack
  - Drains remain partially effective even if penetrated by a horizontal crack as evidenced from research by University of Colorado
- If the evaluation indicates the section has cracked all the way through (limiting case), you should consider uplift pressures no greater than those associated with tailwater at the downstream face

Beware of Apparent Cohesion on Cracked Section

Strength is often over-estimated by straight line fit (at low normal stress typical of gravity dams)



Gravity Dams – Shear Strength



- Make sure added strengths are developed at compatible displacements

Leaking Lift Joints

- Not necessarily un-bonded
- Friant Dam – numerous leaking lifts, but core showed them to be intact
- Check construction records to get a sense for how likely the joints are to be bonded
- Good joint treatment would include water curing tops of lifts, green-cutting (or sand blasting) laitance, and richer mix/smaller aggregate on top of cured concrete

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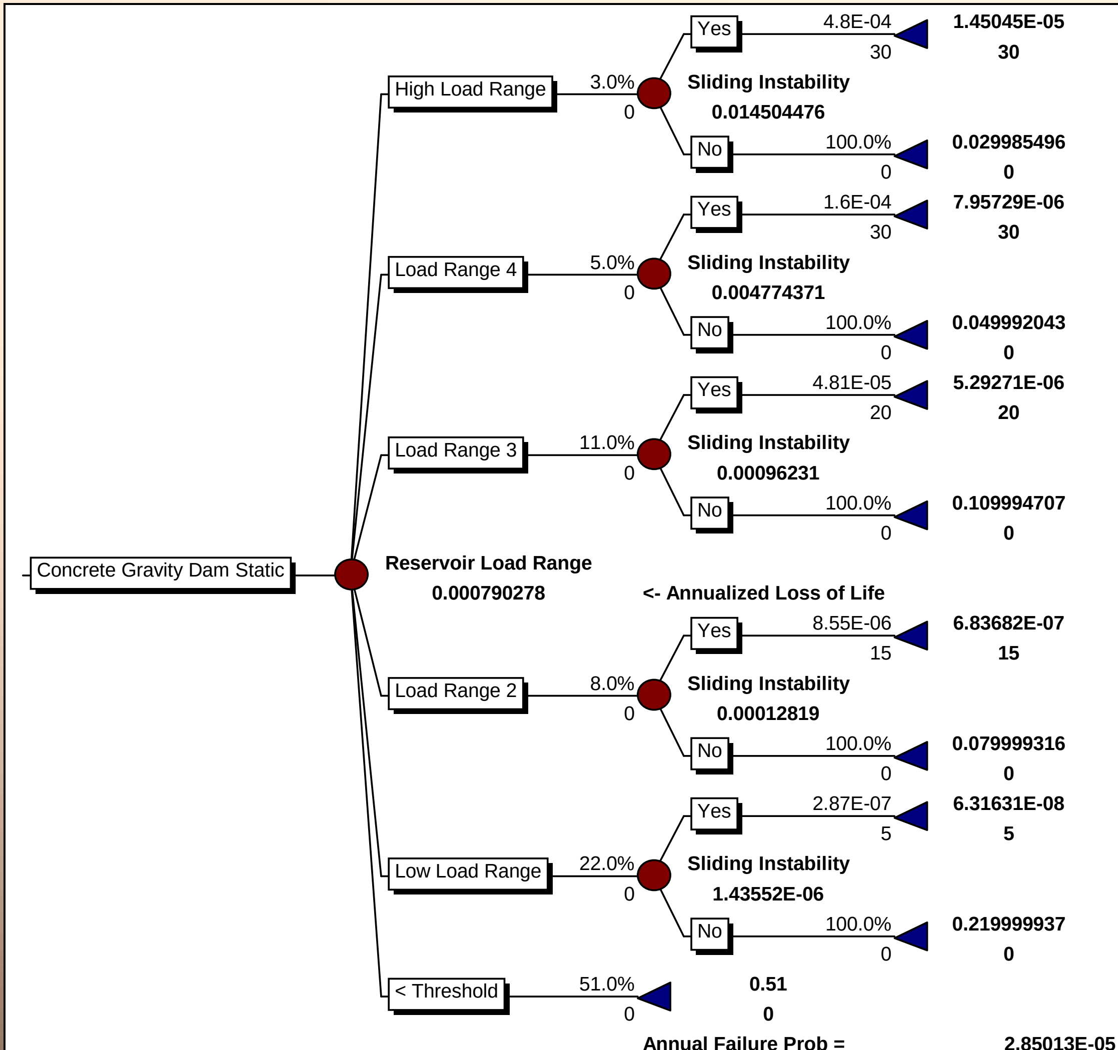
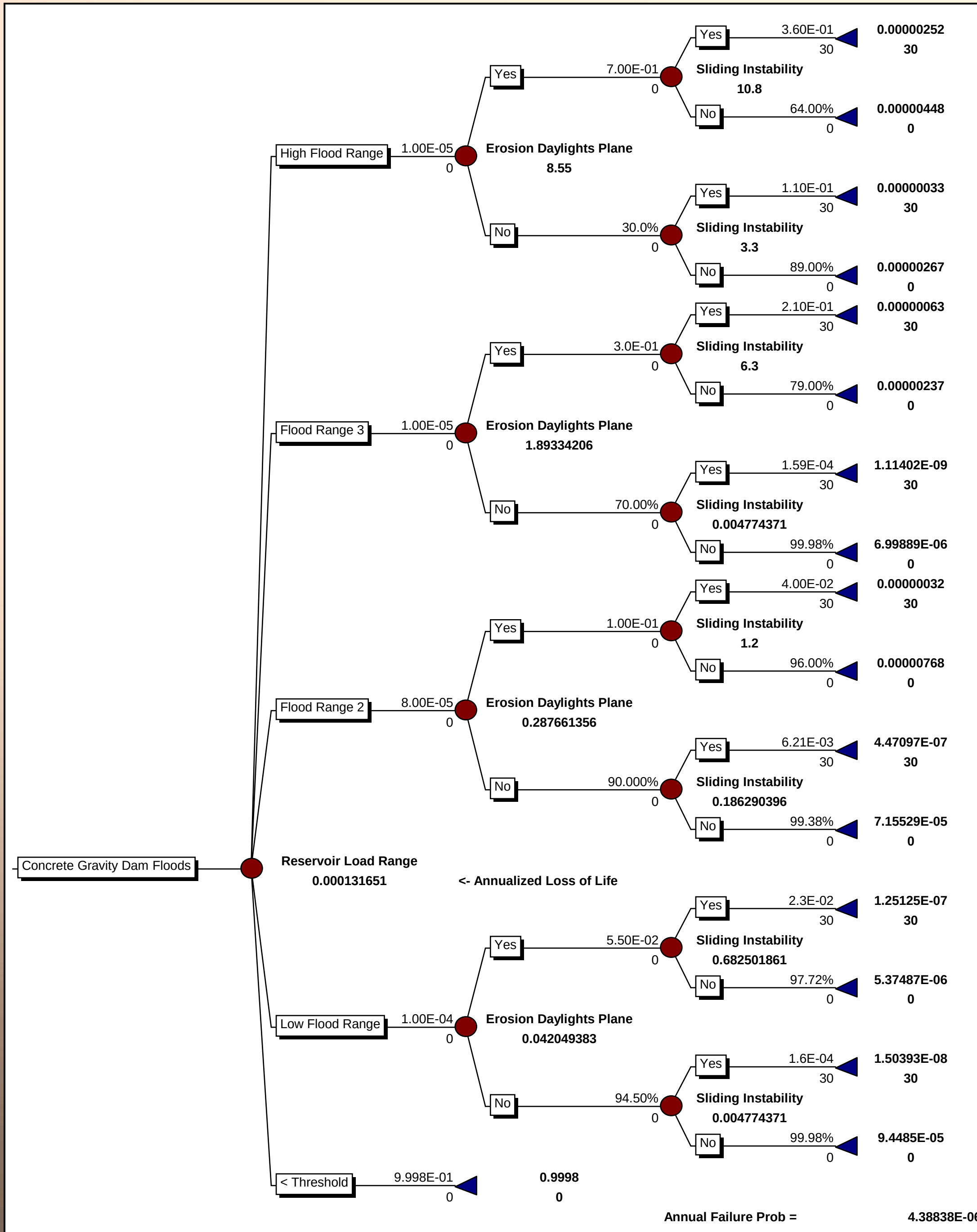
Risks Under Normal/Flood Loading Only



- Use reliability analysis results for 2-D analysis sections and various loads to provide basis for risk estimates
- Account for any 3-D effects judgmentally (as part of elicitation methods)
- Prudent to use tighter ranges around critical load levels (excessive heel stresses, etc)
- Use as input to event trees for risk estimate



Other Considerations

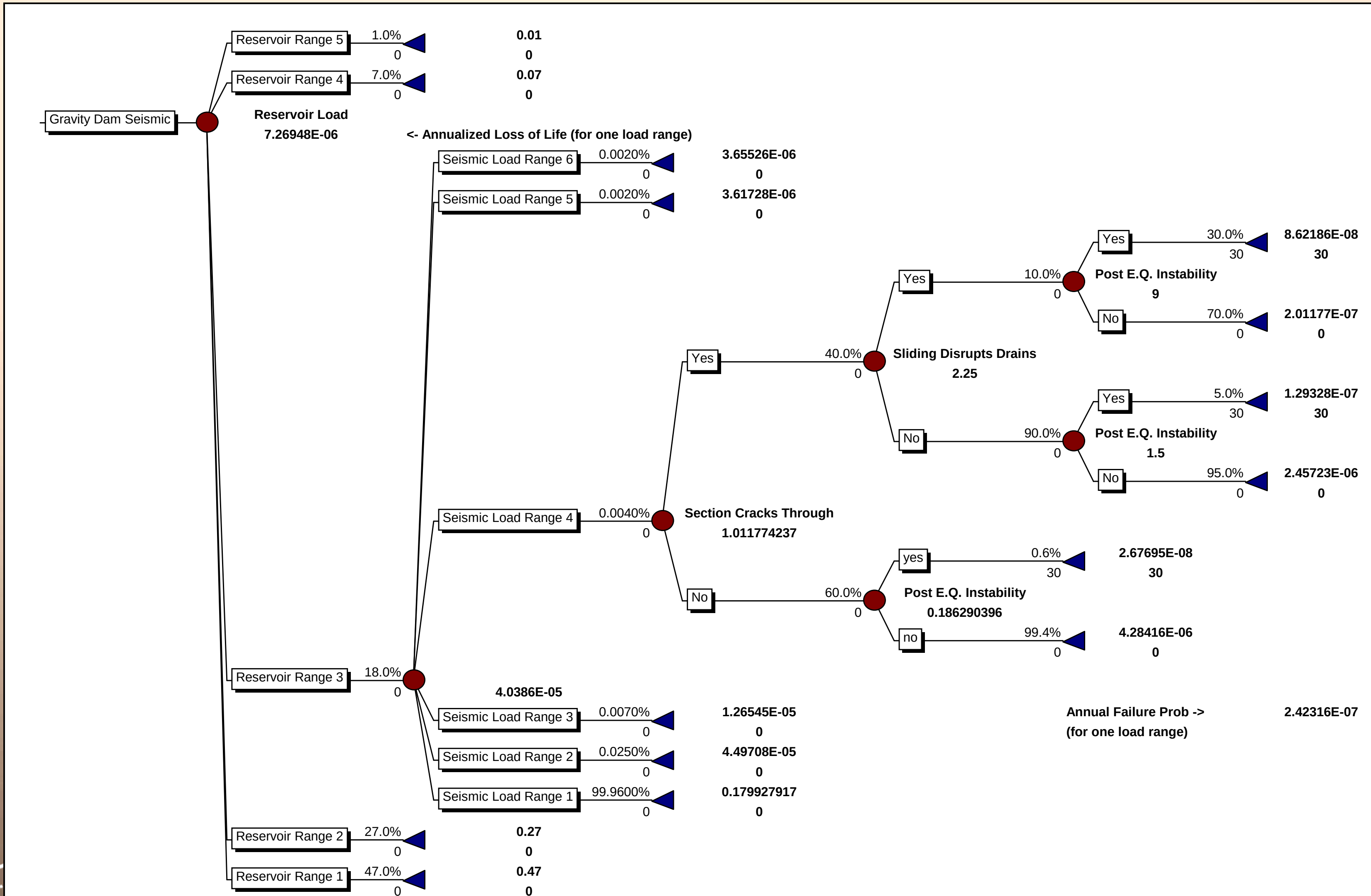
- Careful of nappe and tailwater forces
- Pay attention to potential erosion of rock providing passive resistance (Is there sufficient duration of spillway releases?)
- Will a erosion open (daylight) a weak plane?
- Fully develop event tree and estimate through combination of analytical and elicitation methods

Seismic Risks

Things to Consider Evaluating for EQ Loading

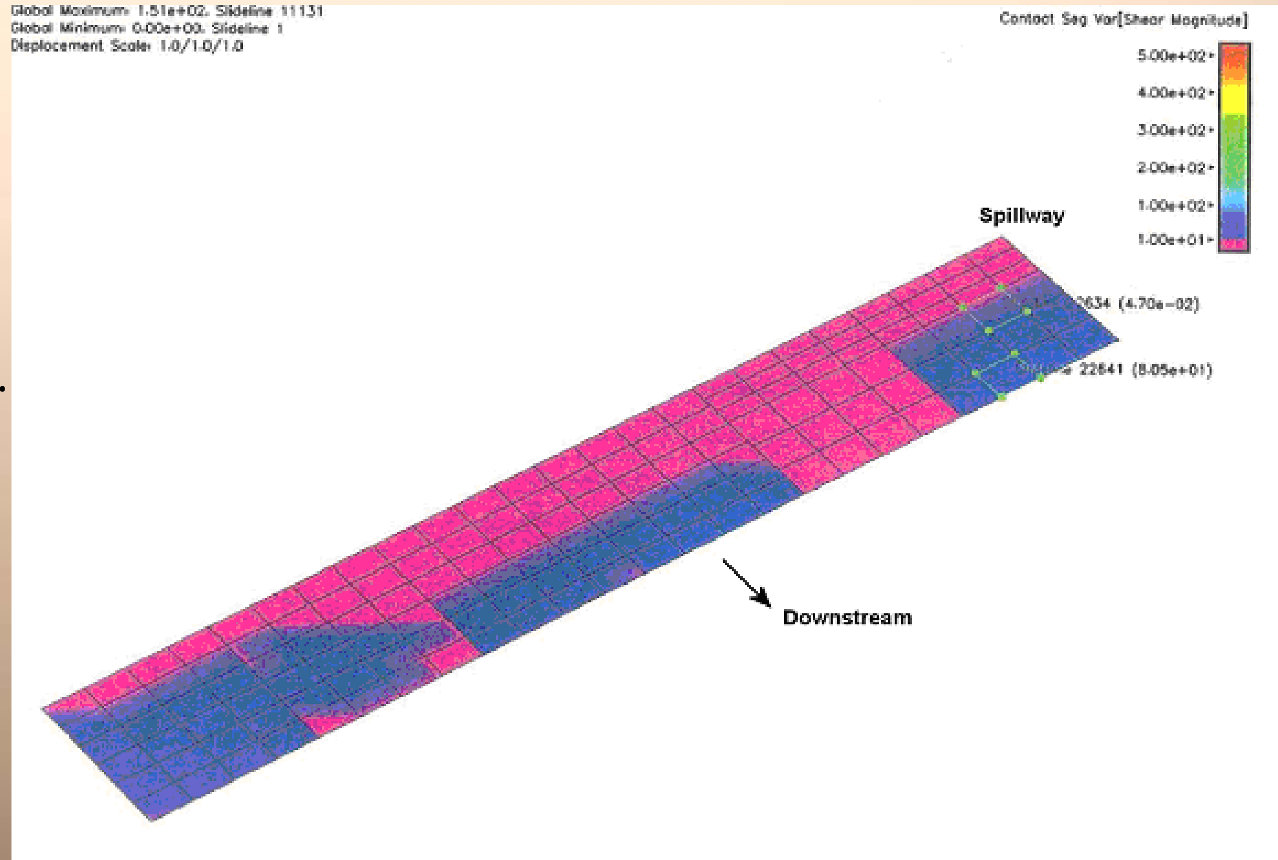
- Likelihood of cracking through the section
- Likelihood of sufficient displacement to displace drains and increase uplift
- Likelihood of post-earthquake instability
- Dependent on earthquake load and reservoir level at time of earthquake (frequency needs to be considered in risk analysis)

Seismic Risks



Likelihood Section Cracks Through

- Nonlinear finite element analysis
- 2-D or 3-D



Important Note: Nonlinear analysis is not for the faint of heart. Make sure you thoroughly test your model. Make sure it can give the correct answer to simple problems, etc. Build the case for why someone should believe the results.

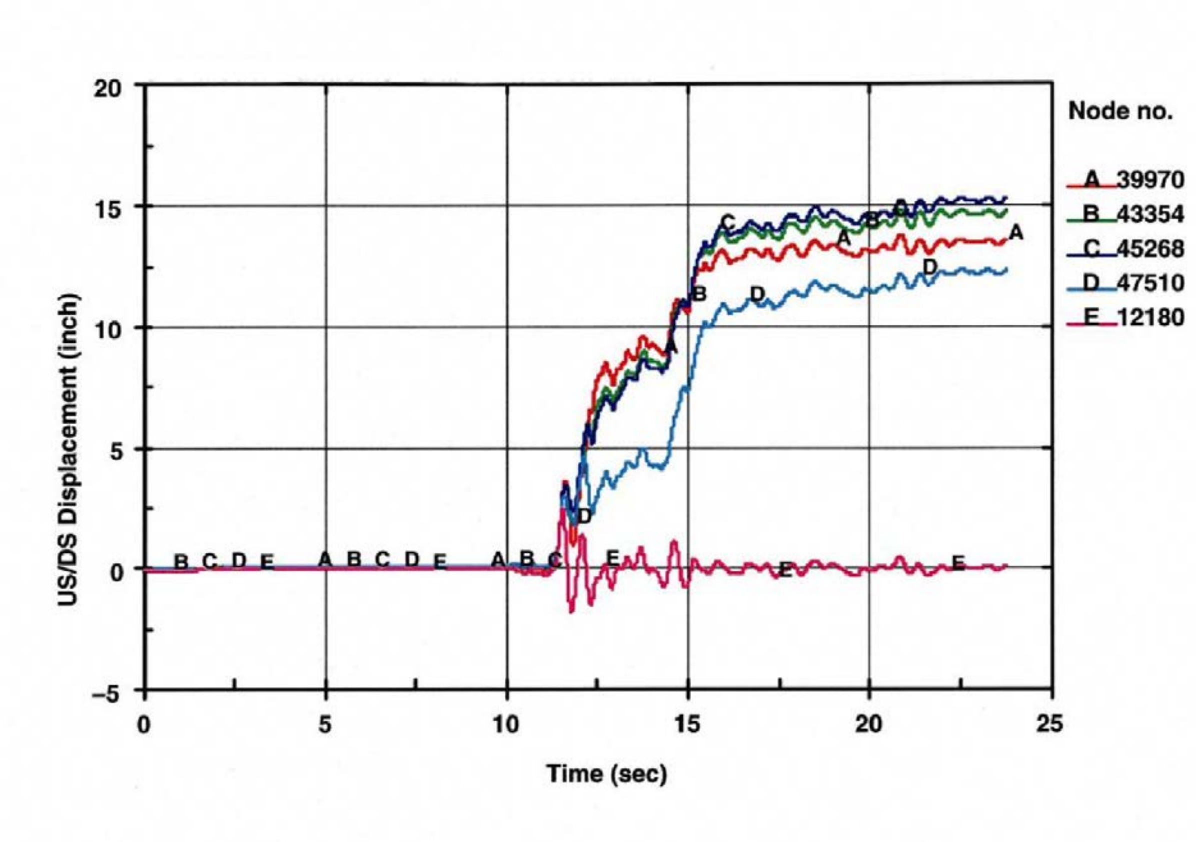
Likelihood of Cracking Through

- Adverse Factors
  - Tensile stress on u/s face exceeds estimated dynamic tensile strength for upper load ranges
  - Cracks may propagate more readily than nonlinear analysis accounts for
- Favorable Factors
  - Tensile stress on u/s face is less than estimated dynamic tensile strength for most load ranges
  - Coring showed good bond at lift joints
  - Nonlinear analysis showed only one monolith would crack through at upper load range
- Identify Key Factors and Build the Case for Estimate

Likelihood of Shearing Drains

- Nonlinear finite element analysis
- Unbonded surface or shear/tensile cutoff

Some simplified equations included in manual for estimating displacements from yield acceleration – use with caution



Likelihood of Displacement/ Increase in Uplift

- Adverse Factors
  - Nonlinear analysis showed displacements greater than drain diameter at upper load range.
  - Dilation on sliding plane could increase uplift without displacing drains
- Favorable Factors
  - Nonlinear analysis showed displacements less than ½ the drain diameter for most load ranges
  - Nonlinear analysis assumed lift was cracked at beginning of E.Q. when in fact it is bonded
  - Nonlinear model did not include embankment wrap-around which could reduce sliding at ends, causing rotation and binding at contraction joints
- Identify Key Factors and Build the Case for Estimate

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Likelihood of Post Earthquake Instability

- Use reliability analysis of damaged section for various scenarios
  - Partially cracked section
  - Fully cracked section but drainage intact
  - Fully cracked section with drains sheared

Exercise

- Given: The upper 34.4 feet of a concrete gravity dam above a lift joint with base thickness of 16.2 feet and a reservoir loading of 32.6 feet above the base; Calculate the total stress and effective stress at the upstream face in this location. The weight of this section of the dam is 64.4 kips/ft, and the moment induced by the reservoir load on the upstream face and the dam weight together is 279 kip-ft/ft (downstream rotation). (The moment of inertia is equal to the base thickness cubed divided by 12.) Is the dam likely to crack at this location if it is constructed of conventional concrete with an unconfined compressive strength of 3,500 lb/in2 and 6-inch maximum size aggregate?

Possible Exercise Solution

Weight =	64,400	lbf/ft		
Moment =	279,000	lbf-ft/ft		
Thickness =	16.2	ft		
Moment of Inertia =	354.29		B3^3/12	
Water Depth @U/S face =	32.6	ft		
Water Pressure @ U/S Face =	2034.24	psf	B5*62.4	
Negative stress tensile				
Stress @ Heel (no uplift - total stress)	-2403.29	psf	B1/B3-B2*(B3/2)/B4	
Stress @ Heel (with uplift - effective stress)	-4437.53	psf	B8-B6	
Converted to psi =	-30.82	psi	B9/144	tension
Concrete Compressive Strength =	3,500	psi		
Tensile Strength =	391.89	psi	1.7*B12^(2/3)	
Reduce for large aggregate	352.70	psi	B13-0.1*B13	
Reduce for vertical strength	282.16	psi	B14-0.2*B14	
Reduce for lift joint	239.83	psi	0.85*B15	